Technical Report REMR-GT-22
June 1995

US Army Corps of Engineers
Waterways Experiment Station

Repair, Evaluation, Maintenance, and Rehabilitation Research Program

The State of Practice for Determining the Stability of Existing Concrete Gravity Dams Founded on Rock

by James K. Meisenheimer

Prepared for Headquarters, U.S. Army Corps of Engineers

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	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

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Rehabilitation Research Program

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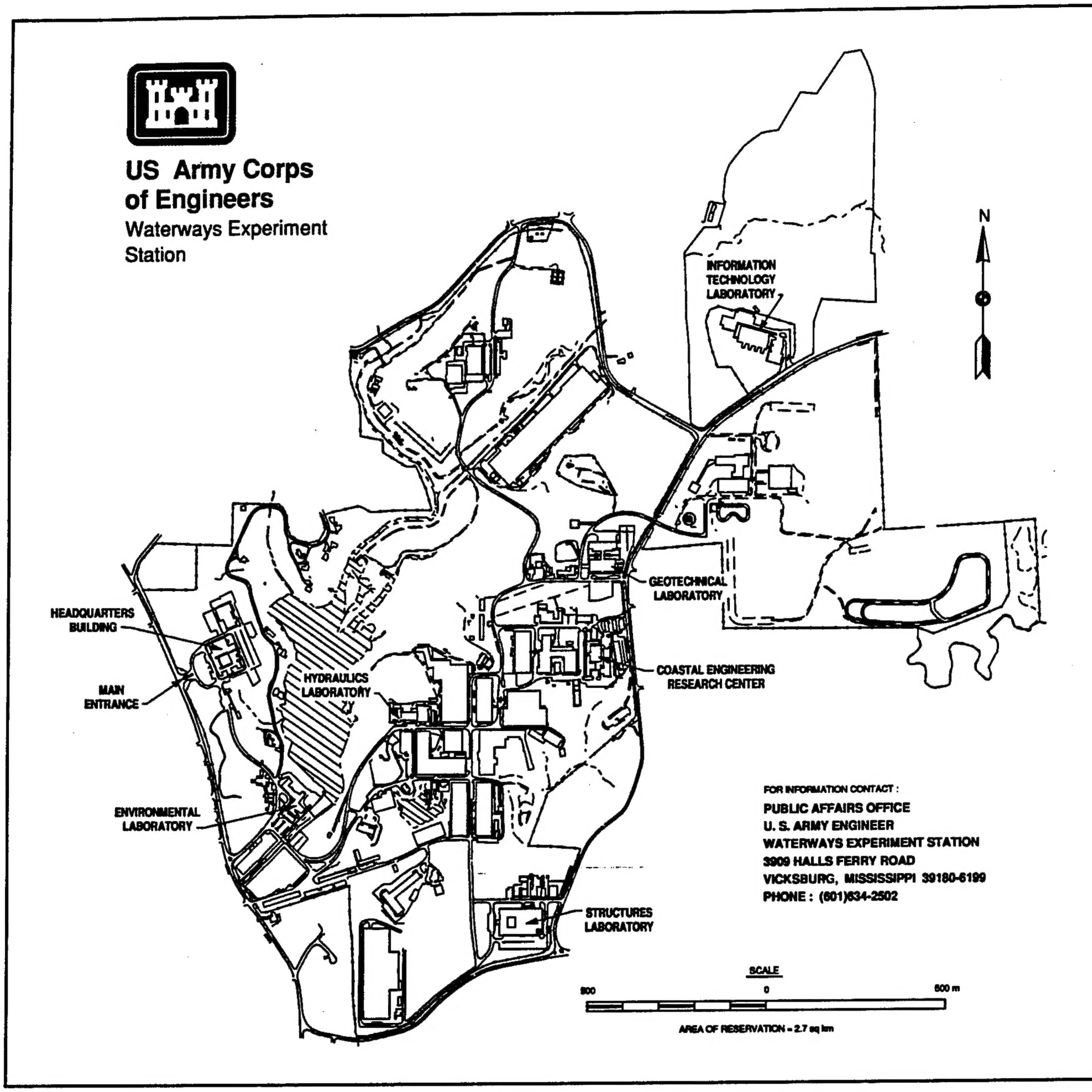

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Contents

Preface

The work described in this report was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Geotechnical Rock Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. The work was performed under Work Unit 32648, "Geomechanical Modeling for Stability of Existing Gravity Structures," for which Mr. Robert D. Bennett, Soil and Rock Mechanics Division (S&RMD), Geotechnical Laboratory (GL), U.S. Army Engineer Waterways Experiment Station (WES), was Principal Investigator. Mr. Wayne Swartz (CECW-EG) was the REMR Technical Monitor for this work.

Mr. William N. Rushing (CERD-C) was the REMR Coordinator at the Directorate of Research and Development, HQUSACE; Mr. James E. Crews (CECW-O) and Dr. Tony C. Liu (CECW-EG) served as the REMR Overview Committee; Mr. William F. McCleese, WES, was the REMR Program Manager. Mr. Jerry S. Huie, S&RMD, was the Geotechnical Rock Problem Area Leader. The project was under the general supervision of Dr. Paul F. Hadala, Assistant Director, GL, and Dr. William F. Marcuson III, Director, GL.

This report was prepared by Mr. James K. Meisenheimer, Stone and Webster Engineering Corp. Mr. Bennett was the Contracting Officer's Representative. Dr. Ronald B. Meade (GL) served as Assistant Principal Investigator.

At the time of publication of this report, Dr. Robert W. Whalin was Director of WES. COL Bruce K. Howard, EN was Commander.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
feet	0.3048	meters
inches	2.54	centimeters
kips (force)	4.448222	kilonewtons
kips (force) per square foot	47.88026	kilopascals
kips (force) per square inch	6.894757	megopascals
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
square feet	0.09290304	square meters
tons (force) per square foot	95.76052	kilopascals

1 Introduction

Background

To appropriately model and analyze the static stability of existing concrete gravity dams, it is necessary to know the hydrostatic uplift pressures acting on a dam and the interaction of the concrete and rock with respect to behavior and strengths. Knowledge of the characteristics of the geologic structure beneath the dam, such as the fracture and joint configuration, is important in defining seepage flow and weak areas that might be critical to sliding stability. The geologic information is also used to properly define and perform a realistic stability assessment. However, these detailed geologic and strength data are generally lacking, especially for older dams, and not readily measurable due to the physical presence of the existing dam.

Older concrete gravity dams were initially analyzed and designed for a much smaller design flood than presently required. Uplift was not generally considered to act over the entire base, and rock strength was often overvalued in the original analyses. Existing dams are generally reanalyzed under present day loading requirements by performing a preliminary assessment to evaluate stability. This assessment uses conservative assumptions for uplift pressure distribution, definition of sliding planes, and frictional resistance along the sliding planes. Based on this approach, several of the U.S. Army Corps of Engineers' (CE) older concrete gravity structures fail to meet the current stability criteria for sliding and overturning, and expensive rehabilitation appears to be the only solution for stabilization. However, it is generally discovered after further review that even though the conservative assumptions and parameters used indicate that the historical floods of record should have caused the dam to fail, the dam is still performing satisfactorily. Most existing concrete gravity dams have generally experienced severe flood loadings and performed well. Dam history and performance provide valuable information that should be included in the reevaluation process.

Expensive rehabilitation schemes have recently been undertaken to stabilize some concrete gravity structures. Many of these activities have been based on a simplified analytical approach, without a clear understanding of the dams' actual behavior. On the other hand, the performance and behavior of other older dams have recently been thoroughly reassessed and reevaluated with respect to foundation geology and uplift conditions. For these dams, cost-effective rehabilitation programs have been developed that are considered

satisfactory to improve stability. Some important questions to be answered for any stability analysis are whether geologic, geometric, and boundary conditions are being accurately and appropriately used to define failure surfaces and uplift conditions.

Problems Evaluating Existing Gravity Dams

The most difficult task in evaluating existing concrete gravity dams lies in obtaining a realistic assessment of the geologic and uplift conditions that exist beneath the dam. Detailed geologic investigations and testing were generally not performed for older dams. Records of dam construction and foundation preparation and treatment may not be available. Detailed exploration programs to evaluate an existing concrete gravity dam are very difficult to execute, expensive, time consuming, and usually limited in the amount of meaningful data that can be obtained. Additionally, there are limited databases assembled that can be used to provide guidance to assist in defining the expected behavior and performance of the various geologic conditions that can occur beneath existing concrete gravity dams.

While evaluating the performance and behavior of numerous dams for the Electric Power Research Institute (EPRI) Concrete Gravity Dam Stability Program (EPRI 1992) trends in dam movement and uplift were noticed that were characteristic of other sites' geologic conditions. Also, some studies with instrumented navigation locks, which are smaller gravity structures, showed promise in providing an additional data resource for understanding dam foundation interaction. Locks exist on geologic units similar to some of the questionable CE dams. Observing lock performance with varying headwater loadings is another avenue that expands the understanding of dam foundation interaction and behavior. Since locks are generally parallel to valley sides and gravity dams are perpendicular, implications with regard to geological processes and geotechnical framework, e.g., valley stress relief in foundation geology, has to be considered (Ferguson and Hamel 1981). The possible differences in geometry, loading, and operating conditions must be considered when extrapolating behavior from locks and correlating that to gravity dams.

Objective

The objective of this study is twofold. The first is to document the state of practice in the United States for determining the stability of existing concrete gravity dams under static loading conditions. Considerable work has been accomplished in the last 4 years in evaluating the performance of existing gravity dams, and an assessment needs to be made to define where it has brought the industry. A second objective is to define where the industry needs to move and provide a focus for activities that can be beneficial to the CE in reevaluating the stability of existing concrete gravity dams. The scope of this report has been limited to evaluating the state of practice for dams

owned, operated, or regulated by federal agencies. This report does not evaluate the state of practice for privately owned dams that may only have to meet dam safety criteria of the state in which they are located.

2 U.S. Army Corps of Engineers Stability Design Criteria and Guidance

Design Load Cases

CE currently uses EM 1110-2-2200 U.S. Army Corps of Engineers[1] (USACE) 1958, which lists three static loading conditions that are applicable to evaluating stability for existing concrete gravity dams:

a. *Load Condition II-Normal Operating Level*. Pool elevation at top of closed spillway gates where spillway is gated, and at spillway crest where spillway is ungated. Minimum tailwater elevation for gated and ungated spillways. Ice pressure if applicable.

b. *Load Condition III-Induced Surcharge Condition*. Pool elevation at top of partially opened gate. Tailwater pressure at full value for non-overflow section and 60 percent of full value for overflow sections. Uplift full value for overflow and non-overflow section. Ice pressure if applicable.

c. *Load Condition IV-Flood Discharge Condition*. Reservoir at maximum flood pool elevation. All gates open and tailwater at flood elevation. Tailwater at full value for non-overflow sections and 60 percent of full value for overflow sections for all conditions of deep flow over spillway, except that full value will be used in all cases for uplift computation. Uplift full value for overflow and non-flow section. No ice pressure

The Probable Maximum Flood (PMF) design loading condition would apply to Load Condition IV.

CE has recently revised EM 1110-2-2200 (USACE 1994a), which is scheduled to be issued in late 1994. In this new revision, load conditions are as

[1] U.S. Army Corps of Engineers will be referred to in text as USACE.

follows: Load Condition II above will be classified as Load Condition No. 2 — usual loading condition; Load Condition III will be classified as Load Condition No. 3 — unusual loading condition — flood discharge, with the reservoir at the standard project flood (SPF); and Load Condition IV will be classified as Load Condition No. 7 — extreme loading condition — probable maximum flood (PMF).

Stability Criteria

Stability requirements

The basic stability requirements from EM 1110-2-2200 (USACE 1958) for a gravity dam for all loading condition are:

a. That it be safe against overturning at any horizontal plane within the dam, at the base, or at a plane below the base.

b. That it be safe against sliding on any horizontal plane within the dam, at the foundation, or on any horizontal or nearly horizontal seam in the foundation.

c. That the allowable unit stresses in the concrete or in the foundation material shall not be exceeded.

The proposed stability requirements for a gravity dam are essentially unchanged for the revised EM 1110-2-2200 (USACE 1994a) except that the failure surface in the foundation will be determined by geologic conditions and may not occur as a horizontal or planar surface. Reevaluation of existing dams is covered by Chapter 8 in the revised EM 1110-2-2200 (USACE 1994a). Procedures are provided for evaluating current structural conditions and determining the necessary measures for rehabilitation of existing concrete gravity dams. A more realistic analytical approach can be performed when preliminary analyses indicate that the structures does not meet current criteria and expensive remediation is required. The District, Division, CECW-E, and CECW-O will agree on a plan for a more refined program and analyses.

Factor of safety

From ETL 1110-2-256 (USACE 1981), the minimum required factor of safety for sliding stability of concrete gravity dams and lockwalls for normal static loading, Load Condition II, is 2.0 for the limit equilibrium method of analysis. The factor of safety (FS) is defined as the ratio of the shear strength (τ_F) and the applied shear stress (τ) according to Equation (1).

$$FS = \frac{\tau_F}{\tau} \tag{1}$$

The factors of safety proposed in the revised EM 1110-2-2200 (USACE 1994a) are shown in Table 1. Table 1 represents the factors of safety that currently should be used by the CE since the revision will be issued in late 1994 and will supersede previous references. The Usual Load Condition corresponds to present Load Condition II, and the Extreme Load Condition corresponds to Load Condition IV with a PMF loading.

Overturning criteria

When the resultant of all forces acting above any horizontal plane through a dam intersects that plane outside of the middle third, it is assumed that a noncompression zone will result. It is necessary to determine whether any noncompression areas exist within the concrete dam or in the foundation when evaluating existing concrete dams. In noncompression areas, uplift is generally considered to be 100 percent of the reservoir level and cohesion within the foundation is not allowed for stability analysis. Allowable tension for concrete is shown in Table 1. Table 1 gives the overturning criteria to be used for resultant locations at the base for the revised EM 1110-2-2200 (USACE 1994a).

Methods of Analyses

Overturning stability

The overturning stability is calculated by applying all the vertical forces (ΣV) and lateral forces for each loading condition to the dam and, then, summing the moments (ΣM) caused by the consequent forces about the downstream toe of the dam. The resultant location along the base is:

$$Resultant\ Location = \frac{\Sigma M}{\Sigma V} \tag{2}$$

Sliding stability

The methods of analyses for sliding stability of concrete gravity dams on rock are outlined in ETL 1110-2-256 (USACE 1981) and further described and defined in the proposed EM 1110-1-2908, (USACE 1994b) and the revised EM 1110-2-2200 (USACE 1994a). Stability analysis will primarily be determined by a limit equilibrium method. For this analytical method, the shear force necessary to develop sliding equilibrium is determined for an assumed failure surface. A sliding mode of failure will occur along a

presumed failure surface when the applied shear stress (τ) exceeds the resisting shear force (τ_F).

The potential failure surface will be controlled by the geologic conditions existing within the foundation base and must be defined as accurately as possible for the analyses to represent actual dam stability. The definition of representative failure surfaces beneath an existing dam is considered the most critical stage for stability analysis and is generally the most difficult task to accomplish, due to a lack of site-specific data. The failure surface can be any combination of planar or curved surfaces. A sufficient number of potential failure surfaces (reasonable with respect to known or inferred geologic conditions) should be analyzed to have reasonable confidence that the most critical failure surface has been located.

The CE methods for analysis requires that defined failure wedges be bound by plane surfaces. The critical failure surface with the lowest factor of safety is determined by an interactive process. Only force equilibrium is satisfied in the analysis. The shearing forces acting parallel to the interface of any two wedges are assumed to be negligible; therefore, only the portion of the failure surface at the bottom of each wedge is loaded by the forces directly above or below the failure surface. It is also assumed there is no interaction of vertical effects between wedges. Resulting wedge forces are assumed horizontal. Considerations regarding displacements are excluded from the limit equilibrium analysis. It should be noted that higher factors of safety are typically obtained with methods that consider moment equilibrium as well as force equilibrium (Hamel, Long, and Ferguson 1976).

When the relative rigidity of different foundation materials and the concrete structure influence sliding stability results, a more intensive sliding investigation than a limit equilibrium approach may be used. A three-dimensional (3-D) analysis may be performed if unique 3-D geometric features and loads critically affect the sliding stability of a specific structure. It should be noted that 3-D limit equilibrium stability analysis will give higher factors of safety than two-dimensional (2-D) analysis.

The CE uses the computer program CSLIDE to perform 2-D sliding stability analysis of gravity dams. The program uses the multiwedge system of analysis discussed. Program documentation is covered in ITL-87-5, *Sliding Stability of Concrete Structures (CSLIDE)* U.S. Army Engineer Waterways Experiment Station 1987).

The computer program CDAMS can be used to perform a 3-D stability analysis of concrete dams. A detailed description and more information about use of the program can be found in USACE (1983).

Selection of Design Parameters

General

The objective of the stability analysis is to model the analysis as closely as practical to the anticipated failure conditions expected to occur beneath the dam. Failure modes and mechanisms and planes of weakness must be defined based onsite geologic conditions, foundation material properties, and uplift forces. Available site geological information is evaluated to define planes of potential failure surfaces and the characteristics of these failure surfaces that are pertinent for defining and selecting design parameters. Generally, preliminary stability analyses are performed for existing dams, using conservative assumptions to assess the dam's stability and to identify the design parameters that require special attention and definition for use in the final stability analysis.

Shear strength parameters

The critical material parameters at the failure contacts within rock are the cohesion intercept (C) and the angle of internal friction (ϕ). The C and ϕ shear strength parameters should be determined by an experienced geological or geotechnical engineer since they have the greatest effect on the sensitivity of the stability analysis. When realistic shear strength parameters can be obtained and used in analyses, lower factors of safety can be considered when evaluating the stability of existing dams. The modes of potential failure along the failure plane surfaces are the most important consideration in selecting shear strength parameters. The mechanisms of potential shearing failure must be defined for the rock mass as to whether failure along the defined plane will occur through previously sheared rock, intact rock, clean discontinuous rock, filled discontinuous rock, or a combination thereof.

Design shear strength parameters are determined by empirical methods and by evaluating the results of direct shear and triaxial tests by the Mohr-Coulomb method. In situ testing is expensive and generally limited to special situations that justify the cost. Design parameters must be selected based on testing performed in accordance with conditions that represent the foundation loading conditions. The selection of design shear strength values for rock materials and varying geologic conditions are discussed in detail (Nicholson 1983) and EM 1110-1-2908 (USACE 1994b). These references include recommendations for testing methods, interpretation of results by the Mohr-Coulomb method, and application to the failure mechanisms expected.

Even though the CE guidelines allow the use of cohesion in stability analyses, there appears to be a reluctance to use design values for C in the analysis. This results in a very conservative analysis for most conditions and can be a costly penalty when the dam rehabilitation is designed to satisfy this analysis. Using even small values of C, when justified, has a significant impact on improving the stability results from the analysis. The selection of shear

strength parameters for stability analysis is dependent on the failure mode expected and the shear deformation characteristics considered appropriate. Peak and near-peak level shear strength parameters are appropriate where there has not been previous shearing displacements in the direction of dam loading. Residual or near-residual level shear strength parameters are appropriate where there have been previous shear displacements in the direction of dam loading. Also, irregularities in the concrete-to-rock interface would allow the use of apparent cohesion whenever a sliding plane shears through them. Acceptable factors of safety for analysis of existing concrete gravity dams should be based on the reliability of the shear strength parameters selected, the method of failures expected, and the historical performance of the dam.

Uplift

Uplift assumptions are provided in the revised EM 1110-2-2200 (USACE 1994a) for dams with no drainage or blocked drainage and dams with drainage. Measured uplift can also be taken into account in the analysis. If monitored uplift is less than general guideline assumptions, additional drain efficiency factors can be applied at the drain line. When a dam has a demonstrated history of functional drains, this reduced uplift should be utilized in the stability analysis so as not to add unnecessary conservatism to the analysis. When a noncompression area exists in the foundation, uplift is assumed to be 100 percent of headwater pressure in this area. When a noncompression area extends beyond the drains, the drain effectiveness shall not be considered. This later uplift loading assumption is generally very conservative and may overly penalize the stability analysis. When an existing dam is being investigated, the design office must submit a request to CECW-ED for a deviation if expensive remedial measures are required to satisfy this later uplift loading assumption.

3 Summary of Recent Advances in the Electric Power Research Institute Concrete Gravity Dam Stability Program

Introduction

The EPRI report (1992) presents the results of a three-year study of uplift pressures and strengths that were used for stability evaluations of existing concrete gravity dams in the United States. Volume 1 of the EPRI (1992) presents the results and conclusions of the study. Volume 2 of the EPRI report (1992) presents details for each host dam, including project design and construction, site description, instrumentation and uplift monitoring data, and a bibliography of technical references and drawings used. Since this report is available to the CE, the extensive reference list and detailed information for each host dam are not presented in this report.

Seventeen host dams were selected and evaluated in detail. These dams were considered representative of the ages, heights, and foundation conditions typical to other existing dams that need to be reevaluated for increased design loadings such as the PMF. Dams with large annual variations in reservoir level change were monitored on a more frequent basis to better define the rate of uplift response. The host dams were built between the period of 1912 and 1974 and ranged in height from 94 ft to 564 ft. A wide variety of sedimentary, metamorphic, and igneous rock foundation conditions were represented. Table 2 lists the host dams and summarizes the EPRI-sponsored work at each dam.

Detailed investigations included core drilling, downhole camera logging of borings and drain holes, piezometer installation, laboratory testing, and evaluation of detailed uplift data obtained over a 1- to 2-year period. The uplift pressures and strengths were shown to be very site-dependent. The EPRI Report (1992) presents a comprehensive approach to stability evaluations that uses site-specific uplift pressures and strengths. Dam behavior is best

understood by observing dam performance and correlating that with other detailed information obtained from site investigations. Defining site conditions with respect to observed dam performance reduces the amount of uncertainty and thus the conservatism that affects engineering judgment required to evaluate stability for future loading conditions such as PMF. Good engineering judgment combined with a focused site investigation program can minimize or avoid expensive rehabilitation repairs to improve stability.

Uplift Response

The EPRI report (1992), Volume 2, presents a comprehensive discussion and presentation of uplift at existing concrete gravity dams. Section 4 of the EPRI report (1992) discusses in depth the effects of geology, foundation treatment, and drainage on uplift pressure distribution beneath the dams, and includes numerous illustrative examples of actual uplift response from host dams.

Influence of geology

The location, permeability, and interconnection of rock discontinuities controls flow through the foundation. The permeability of the discontinuities is generally much higher than that for the intact rock and is thus the major control for uplift below a dam. Because the different types of discontinuities have known effects on uplift pressures, it is possible to predict potential uplift conditions when there are reasonable similarities in site geology. When likely uplift problems have been identified in advance, a site exploration program can be designed more effectively.

Rock discontinuities such as joints, faults, and shear zones are usually several orders of magnitude more permeable than the intact rock. Therefore, these features control the distribution of uplift pressure in the foundation. The degree of interconnection of the joint network influences uplift. In a poorly interconnected network of joints, it is possible that joints fed by the reservoir may not be connected through other joints to tailwater. In this situation, the uplift pressure distribution can be considerably higher than the headwater to tailwater linear distribution usually assumed for dams without drainage curtains. When a highly interconnected joint network is present beneath the dam, the uplift pressure distribution will generally approach a straight line distribution from reservoir level to tailwater. Stress relief effects in valley bottoms often result in foundation rock that is well-drained beneath dams, especially when a well-planned drainage curtain is installed and maintained. This favorable drainage helps explain the stability of many older dams on rock foundations and also the efficiency of drainage for improvement of foundation sliding stability (Cornish and Moore 1991).

Shear and fault zones and clayey bedding planes and seams are planer discontinuities that can have a significant impact on the uplift pressure below

segments of a dam. Broken rock on either side of the shear or fault can have a localized high permeability while the shear zone can have a very low permeability. A low permeable shear zone or low permeable clayey seams can act as an impermeable barrier, resulting in high uplifts beneath and upstream of the dam. This is a very critical design criteria to be evaluated for these situations since these weak zones are generally one of the potential failure surfaces. An important consideration is that the zone of influence of the higher pressures is generally only in the immediate vicinity of the discontinuity and can many times be reduced by installing a proper foundation drainage curtain.

Grout curtains and drains

The case studies also showed that single line grout curtains have no significant effect on uplift. The grout curtains may reduce flows so that the drains work more effectively, but they did not measurably affect uplift. The study results show that drains are, by far, the most effective means to control uplift pressures. Drains can be drilled to provide a highly permeable path between the water-bearing discontinuities. Drainage systems can be designed for variety of uplift controls. This can be accomplished by upgrading drainage in existing dams or installing an uplift drain system in older dams. Drains have proved to be a reliable and cost-effective form of uplift control, especially when monitored and cleaned periodically. When uplift monitoring programs indicate that the drainage system is working effectively to reduce uplift, credit for the reduced uplift should be taken into account when reevaluating stability. Installing drainage systems to reduce uplift has also been used successfully in combination with anchors to stabilize dams at a much reduced rehabilitation cost.

Another aspect of the case studies was to evaluate whether uplift pressures, measured during extreme headwater changes, could be extrapolated to realistically estimate uplift pressures that might occur during a flood. Section 9 of the EPRI report (1992) presents the results of some case histories and a methodology that might be applied. The quality data from these studies support a strong case to the validity of the methodology described in the report. The Federal Energy Regulatory Commission (FERC) will soon allow application of this method when appropriate data are available.

Strength of Concrete Lift Joints

Tensile strength of concrete lift joints and rock contacts

Tensile strength exists in the concrete and in the foundation of a gravity dam. This tensile strength resists the tendency for the force exerted by the water in the reservoir to overturn the structure. Tension in the concrete of the dam is allowed when the lift lines are shown to be bonded and intact. Joints in the rock foundation are commonly assumed to have zero tensile strength; however, blocks of rock in a dam foundation can resist upward tensile forces through self weight and interlock with other blocks. Bonding of the dam and

rock can be demonstrated for many dams but is not generally taken into account in stability analysis as a form of tension resistance force. The EPRI report (1992) summarizes tensile strength data from tests performed on concrete lift joints and concrete-to-rock contacts. The splitting tension test and the direct tensile strength test were compared in the study. The evaluation concluded that only direct tensile strength tests are appropriate for planar features such as lift joints and concrete-to-rock contact. Splitting tests fail along a longitudinal plane, which rarely coincides with the feature to be tested, and therefore are not considered a representative test.

Strength of concrete lift joints

In older dams, the concrete lift lines often have lower bond strength due to construction techniques used at the time and must be evaluated during a stability analysis. To provide a comparative database, direct shear tests were performed on samples obtained at lift joints. Approximately 223 core samples of concrete lift joints from 10 dams were tested in direct shear. The concrete lift line in 69 core test samples was still bonded (uncracked) when tested. The remaining samples were unbonded (cracked) at the time of testing. The majority of the samples cracked during coring or handling and were considered mechanical breaks. All direct shear testing was performed to determine shear strength properties at the lift line locations. The best fit Coulomb strength line for these samples has a peak shear strength friction angle of 57 deg and a cohesion value of 310 psi. Ninety percent of the data points lie above a line with a cohesion intercept of 140 psi and a slope of 57 deg. The sliding friction shear strength data unbonded joints had a best fit friction angle of 49 deg and a cohesion intercept of 70 psi. Ninety percent of the data points lie above the lower bound line through the origin and have a slope of 48 deg. Section 12 of the EPRI report (1992) presents this analysis in detail.

Direct Shear of Concrete-to-Rock Contact

The concrete-to-rock interface is generally one of the potential failure surfaces to be evaluated during stability analysis of a dam. Therefore, representative shear strength parameters are needed in the analysis for the concrete-to-rock interface. Care in sampling, handling, and testing the core sample is important to obtain meaningful results.

Direct shear strength data for concrete-to-rock contacts are available for 18 dams. Approximately one-half of the dams are owned by the CE and the remainder are owned by electric utilities. The dams were built between 1912 and 1965. Direct shear tests for 65 samples of concrete-to-rock contacts were evaluated. Prior to testing, 35 samples were uncracked at the contact, 1 sample was cracked, 11 samples were saw cut, and 18 samples were bonded concrete cast on rock core. Eight different rock types were direct shear tested to determine the concrete-to-rock contact for peak and residual shear strengths. Peak shear strength was determined by the maximum shear load

achieved during the shear test. Residual strength determination is subjective, requiring interpretation of laboratory data to determine when residual strength occurs. For this reported published data, the judgment of experienced investigators was relied on in selecting residual strength. For this study, the residual strength was taken as the lowest consistent set of five or more readings at a displacement between 0.1 and 0.5 in. The shear strength data are presented as the best fit and lower bound Coulomb strength lines drawn on the data plots. Details of this analysis are presented in Section 12 of the EPRI report (1992). The summary of concrete-to-rock contact peak shear strength is presented in Table 3. The concrete-to-rock residual strength summary is presented in Table 4. These tables were developed based on the limited data available during the EPRI study and are not considered to be conclusive, but only a tool to start a reference database.

4 Federal Energy Regulatory Commission Stability Design Criteria

General

The Federal Energy Regulatory Commission (FERC) issued a revised edition of the *Engineering Guidelines for the Evaluation of Hydropower Projects* in April 1991 (FERC 1991). These guidelines have been prepared by FERC's Office of Hydropower Licensing to provide technical guidance in preparing and processing applications for staff evaluation of existing projects and proposed changes or additions to existing projects. They are not intended to provide definitive or specific analytical techniques for engineering analyses but only guidance and examples for analyses.

The dam safety and stability analysis aspects of FERC-regulated projects are administered by the Division of Dam Safety and Inspection. The Division is a very active participant and technical sponsor in the EPRI Concrete Gravity Dam Stability Program. The Division is also an active participant in the U.S. Committee on Large Dams (USCOLD) and other intergovernmental and agencies' meetings on dam safety. The status of the stability of an existing concrete gravity dam that is being reevaluated can be resolved less conservatively than in the past, if well-founded and technically supported analyses are submitted. The Division is allowing recent lessons learned and new methods of analyses, such as from the EPRI programs, to be implemented in evaluating existing gravity dams.

Design Load Cases

The static design load combinations required by FERC for analyses of dams are presented as follows:

a. *Case I-Usual loading combination - normal operating condition.* The reservoir elevation is at the normal power pool, as governed by the crest elevation of an overflow structure or the top of the closed spillway

gates, whichever is greater. Normal tailwater is used. Horizontal silt pressure should also be considered, if applicable.

b. *Case II-Unusual loading combination - flood discharge.* For high and significant hazard potential projects, the project inflow design flood up to and including the PMF, if appropriate, that results in reservoir and tailwater elevations that exert the greatest head differential and uplift pressure upon the structure should be used. However, unusual conditions such as high tailwater should be examined on a case-by-case basis since it is possible that the worst case loading condition exists under other than extreme floods. For dams having a low hazard potential, and which are 25 ft or more in height, or have a storage capacity in excess of 50 acre-ft, the project identified design flood (IDF) considered in the above paragraph should have a frequency up to and including the 100-year flood.

These design load cases are basically the same as presented by each federal agency.

Stability Criteria

General

Stability criteria are dependent on the particular loading combinations being considered, the type of analysis of being performed, and the degree of understanding of the problem, such as the interaction and performance of an existing concrete dam and its foundation. When the unknowns associated with the preliminary analyses and designs are reduced by the final analyses and design stage, FERC may consider lower factors of safety to be acceptable. In the absence of available site data for older existing projects, FERC expects site investigations to be conducted to verify all critical assumptions.

Requirements for static analysis

The criteria and procedures used to conduct stability analyses of concrete gravity dams on rock by FERC are a combination of CE and Bureau of Reclamation requirements, with some modifications based on FERC experience with existing dams. The basic requirements by FERC for stability of an existing gravity dam subjected to static loads are:

a. The moment equilibrium must be maintained on any horizontal plane within the dam, at the base, or at any plane below the base. This requires that the allowable unit stresses established for the concrete and foundation materials are not exceeded. The allowable stresses should be determined by dividing the ultimate strength of the materials by the appropriate factors of safety in Table 5.

b. That it be safe against sliding on any horizontal plane within the dam, at the dam-foundation interface, or on any horizontal seam in the foundation.

Tensile stresses

The tensile stresses within the body of the dam are not to exceed 10 percent of the compressive strength of the concrete. Tensile strength in older dams may be less at lift lines and have to be established by tensile testing. Tensile strength at the rock-concrete interface is assumed to be zero by FERC. Even if there is a good bond between the concrete and the rock, FERC considers that the rock has an unbonded joint or fracture just below the dam.

Cracked base assessment

For existing structures, theoretical base cracking will be allowed for all loading conditions, provided that the crack is stabilized, the resultant of all forces remains within the base of the dam, and adequate sliding safety factors are obtained using cohesion only on the uncracked portion of the base. Any portion of the foundation not in compression is considered "cracked" and is subject to full headwater pressure. Limitations may be placed by FERC on the percentage of base cracking allowed if little foundation information is available or if a highly cracked foundation is indicated. For existing concrete gravity dams, FERC recommends that a cracked base analysis (Bureau of Reclamation 1977) be performed. This report provides a detailed description of this analysis.

Factors of safety

The recommended factors of safety are presented in Table 5. When there is a high degree of confidence in the performance and interaction of an existing dam with its foundation, the design parameters used, and the appropriateness of the analyses, FERC will often accept somewhat lower factors of safety than recommended.

Methods of Analyses

FERC currently allows use of any proven industry standard analytical method that is appropriate. The method of analysis selected and the justification of parameters selected are the responsibility of the dam owners and their engineers and consultants. FERC has the right to disapprove the results or not agree with the design assumptions used. The dam owners and their engineers are responsible for resolving any issues. If there is a particularly difficult problem, FERC may select an independent consultant to provide consultation to them. This is usually beneficial in that it generally involves a

highly respected and very experienced problem solver who helps bring the issue to closure.

The FERC guidelines (1991) provides a very good reference of analytical methods that would be appropriate for the various design cases, with Appendix IIIc in Chapter 3 having several examples of stability analysis using different analytical approaches.

Selection of Design Parameters

General

Design parameter selection is the responsibility of the owners and their engineers. Unless very conservative parameters can be used and still establish that an existing dam is safe, FERC will expect site-specific data to be utilized to support more rigorous and realistic analyses for older dams. Generally, it is difficult to prove that older dams are stable under present day design loadings such as the PMF when only very conservative design parameters are available for use. FERC will allow historical data such as dam performance during floods, site construction photographs, and other historical construction and engineering data to be used with a site investigation program. This has proved to be a cost-effective approach. The critical problems can generally be identified through initial evaluations and isolated to make the site program more focused. The FERC guidelines (1991) provides only references to assist in obtaining background information for developing design parameters. FERC will also make available to owners the results of similar studies and programs that are public record and possibly pertinent to their project.

Uplift assumptions

FERC has very specific criteria relative to uplift assumptions to be used in an analysis of the foundation-concrete interface. When there are no foundation drains or the drains are determined to be clogged, the uplift pressure is assumed to act over the entire base and have 100 percent headwater pressure at the upstream heel and vary as a straight line to 100 percent tailwater pressure at the toe of the dam (Figure 1).

When a dam has operable drains and uplift performance data, the uplift pressure acting on the base will vary from 100 percent headwater at the heel to an established effective uplift level at the drain and the straight line to 100 percent tailwater at the toe of the dam (Figure 2). Drain effectiveness has to be established for the design uplift pressure at the drain line and the owner of the dam must commit to a drain cleaning and maintenance program and a continued uplift monitoring program.

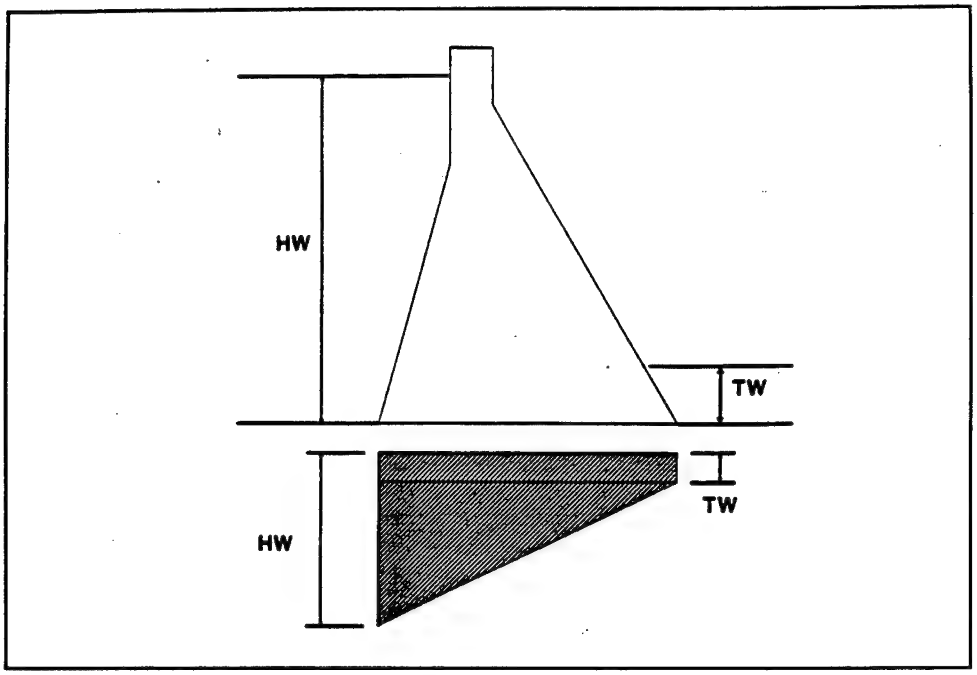

Figure 1. Uplift assumptions with no drainage provided (FERC (1991))

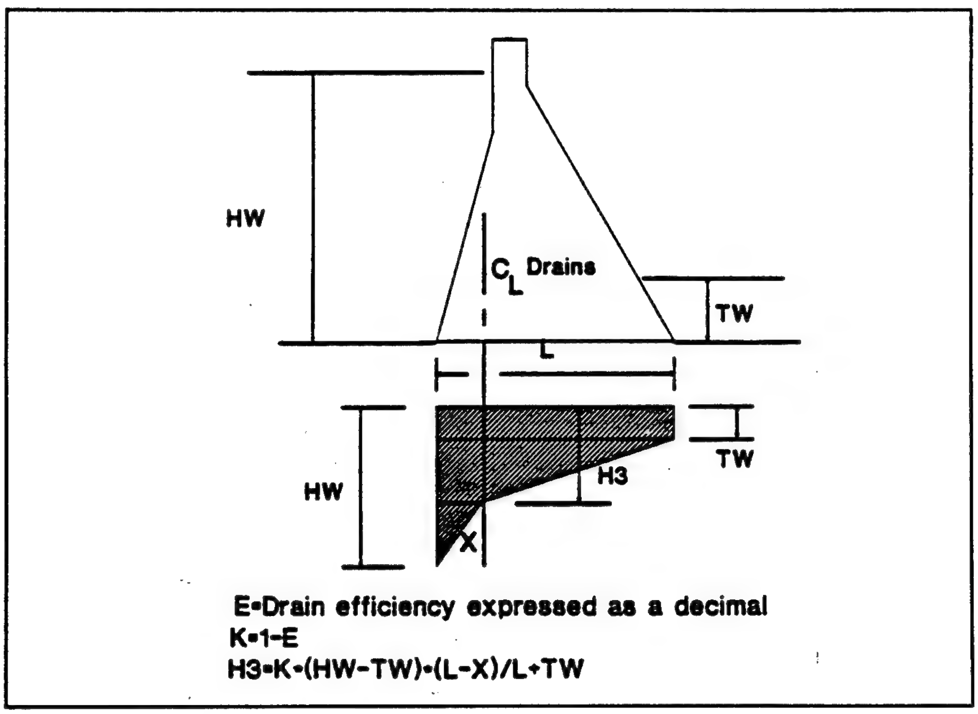

Figure 2. Uplift assumptions with foundation drainage provided (FERC (1991))

In cases where any portion of the base or section of the dam is not in compression, 100 percent headwater pressure is to be assumed for uplift for the noncompression area, unless instrumentation can verify a lower value.

FERC will allow extrapolation of uplift pressures for some flood loading conditions if there is an appropriate supporting uplift database. This will only be done on a case-by-case basis.

5 U.S. Bureau of Reclamation Stability Design Criteria

General

The Bureau of Reclamation evaluates existing concrete gravity dams on a case-by-case basis. Priorities for evaluating existing structures are based on concerns for safety or for planning new modifications. The majority of all studies, investigations, and analyses are performed with in-house engineering staff and augmented, as required, with the use of consultants and contractors. All projects are reviewed by an independent Consultant Review Board. All methods of analyses, design parameters, conclusions, recommendations, and factors of safety are reviewed and concurred with by the Consultant Review Board. The use of a highly experienced staff and a recognized Consultant Review Board provides for innovative approaches to solve particular problems, when the needs arise.

The guidelines for evaluating the stability of existing concrete gravity dams are presented in two design documents (Bureau of Reclamation 1976, 1977). Deviations from these guidelines are permitted when a more appropriate approach and analysis are required and justified to evaluate a particular problem. These referenced guidelines form the basis for discussions in the following sections. Only static design load applications are addressed in this report.

Design Load Cases

General

Designs are based on the most adverse combinations but do include those loads having a reasonable probability of simultaneous occurrence. Temperature loadings are included when applicable.

Usual loading combination

The Usual Loading Combination is based on the normal design reservoir elevation, with appropriate dead loads, uplift, silt, ice, and tailwater. If temperature loads are applicable, the minimum usual temperature is used.

Unusual loading combination

The Unusual Loading Combination is based on the maximum design reservoir elevation, with appropriate dead loads, uplift, silt, minimum temperatures occurring at that time as applicable, and tailwater.

Other

Other load combinations may be included with the above analysis such as drains inoperative, dead loads, any of the above combinations for foundation stability, or other loading combinations the designer considers pertinent to a particular dam.

Stability Criteria

Basic considerations

All safety factors listed are considered minimum values. Somewhat lower safety factors are permitted for limited local areas within the foundation; however, the overall safety factors of the dam and its foundation must meet the requirements for the loading combinations being analyzed. Somewhat higher safety factors are used for foundation studies where there is a greater amount of uncertainty involved in assessing foundation load-resisting capacity.

Allowable stresses

The maximum allowable compressive stress in the concrete for the Usual Loading Combination is not to be greater than the specified compressive stress divided by a safety factor of 3.0. The maximum allowable compressive stress for the Unusual Loading Combination is not to be greater than the specified compressive stress divided by 2.0. The maximum allowable compressive stress is also limited to 1,500 psi for the Usual Loading Combination and 2,500 psi for the Unusual Loading Combination.

The maximum allowable compressive stress in the foundation must be less than the compressive strength of the foundation material divided by safety factors of 4.0 and 2.7 for the Usual and Unusual Loading Combinations, respectively.

Sliding stability

The factor of safety for sliding stability is determined by the shear-friction factor, Q, as described in paragraph 69 using Equation (6). The shear-friction factor is a measure of the factor of safety against sliding or shearing within the structure, its contact with the foundation, or sliding failure along planes of weakness that may exist within the foundation. The minimum shear-friction factor within the dam or at the concrete-to-rock contact should be 3.0 and 2.0 for the Usual and Unusual Loading Combinations, respectively. The factor of safety against sliding on any plane of weakness within the foundation is recommended not to be less than 4.0 and 2.7 for the Usual and Unusual Loading Combinations, respectively. When a computed safety factor is less than required, corrective action is to be initiated to achieve the desired value.

Methods of Analyses

Gravity method of stress and stability analysis

The gravity method of stress and stability analysis is used only for preliminary evaluations of smaller straight dams in which the transverse contraction joints are neither keyed nor grouted. Since most of the Bureau of Reclamation dams are large and located in variable and steep valleys, a more complex method of reanalysis is generally used by the Bureau of Reclamation to determine stresses within the dam and at the foundation, such as 2-D and 3-D finite element analyses. Others in the dam industry use the gravity method of stress and stability analysis. The Bureau of Reclamation (1976) details the equations, approach, and examples for this analysis. Other foundation stability analyses such as 2-D and 3-D finite element methods, differential displacement analysis, and analysis of stress concentrations due to bridging are also presented in the Bureau of Reclamation (1976).

Cracking

Cracking near or at the base of a dam is checked during an evaluation of an existing dam. Higher design water loads and assumptions of clogged or inoperative drainage generally will result in an analysis to determine if a crack might develop. When checking stability with no drainage or inoperative drains, the uplift is assumed to vary linearly from full reservoir level at the upstream face to tailwater level at the downstream face. If cracking occurs in the foundation, full uplift is generally assumed to act across the cracked area, unless uplift monitoring is in place to justify a less conservative condition. Diagrams of base pressures acting on a gravity dam are shown in Figure 3 and are determined by the following procedure from the Bureau of Reclamation (1977):

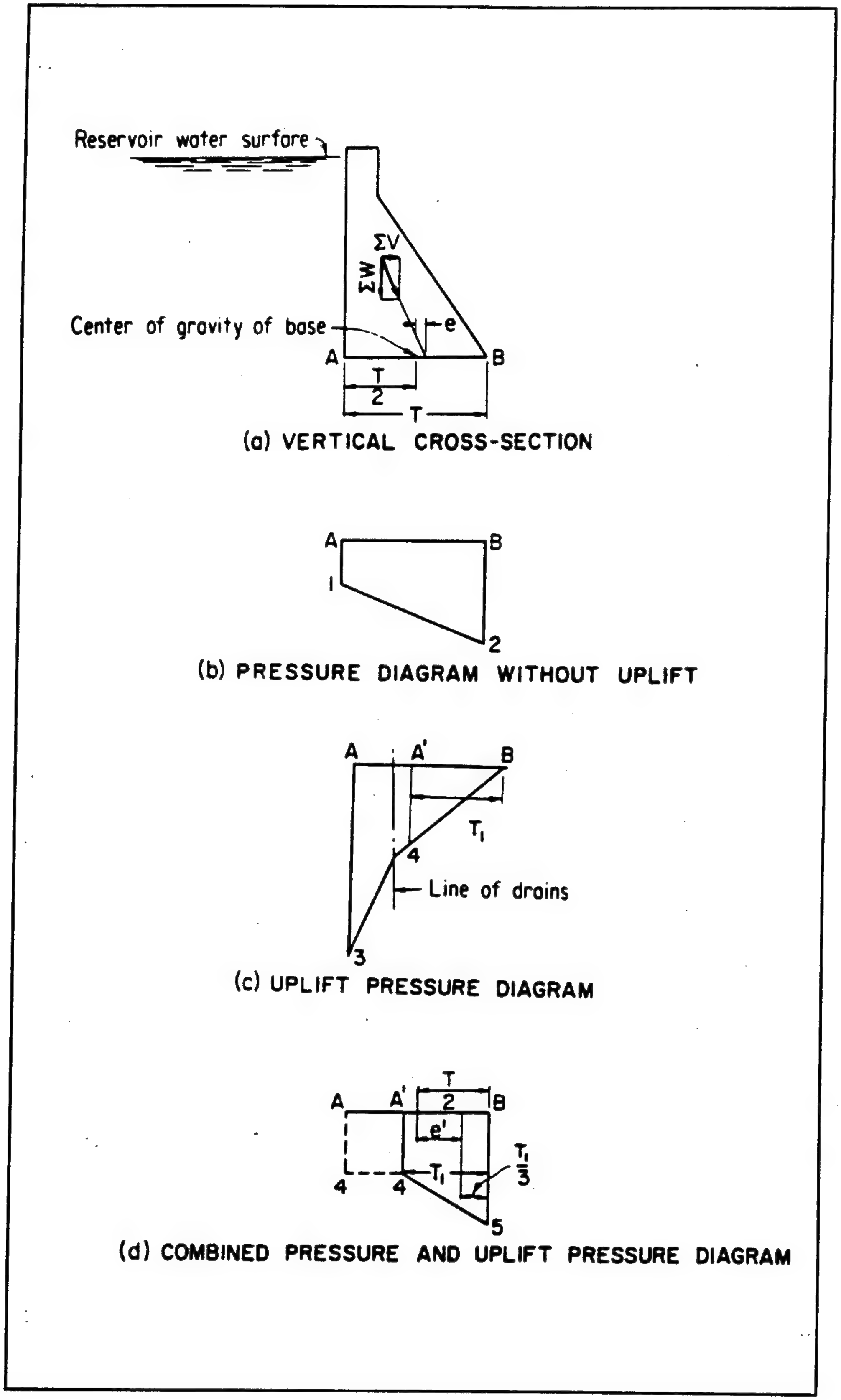

Figure 3. Diagrams of base pressures acting on a gravity dam. Bureau of Reclamation (1977)

a. A horizontal crack is assumed to extend from the upstream face to a point where the vertical stress is equal to the uplift pressure at the upstream face, point 4 in Figure 3(d).

b. From Figures 3(a) and 3(d), taking moments about the center of gravity of the base, the following equations are obtained:

$$e' = \frac{\sum M + M_u}{\sum W - \overline{A'4}\,(T_1)} \tag{3}$$

and

$$T_1 = 3\left[\frac{T}{2} - e'\right] \tag{4}$$

where

e = eccentricity of the stress diagram without a crack

e′ = eccentricity of the stress diagram after cracking

M_u = moment of the tentative uplift force $(\overline{A'4} \cdot T_1)$

ΣM = summation of moments of all forces

ΣW = summation of vertical forces

$\overline{A'4}$ = internal hydrostatic pressure at the end of the crack (A′ in Figure 3(c))

T = the original thickness (Figure 3(a))

T_1 = remaining uncracked portion of the thickness

Therefore the stress at the downstream face of rock $\overline{B5}$, is:

$$\overline{B5} = \frac{2\left[\sum W - \left(\overline{A'4} \cdot T_1\right)\right]}{T_1} + \overline{A'4} \tag{5}$$

The T in Equation (4) is the full thickness of the original section. The value of T_1 to be used in Equation (3) must be estimated for the first computation. Thereafter, for succeeding computations of e', the value obtained for T_1 in Equation (4) should be used. Several cycles of computation using Equations (3) and (4) are required to obtain adequate agreement between the value used for T_1 in Equation (3) and the value computed for T in Equation (4).

The uncracked area of the base is substituted for A in Equation (6) below. The section is considered satisfactory for any of the loading conditions if the stress at the downstream face, from Equation (5), does not exceed the allowable stress, and the shear-friction factor of safety is acceptable. The gravity dam is considered safe against overturning if $\overline{B5}$, the ordinate in Figure 3(d), is less than the allowable stresses in the concrete and foundation rock for the appropriate loading combinations. Equations (3), (4), and (5) and symbols are listed as they appear in the design criteria of Bureau of Reclamation (1977). This method for determining a cracked base length is widely used in the dam industry since it is referenced by FERC (1991) as a criteria that must be checked.

Sliding stability

Foundation sliding stability is computed using the shear-friction method, limit equilibrium, or other 3-D rigid block methods. The limit equilibrium method currently used by the Bureau of Reclamation as the preferred method of sliding stability analysis is the CE method (Nicholson 1983). The Bureau of Reclamation generally uses a finite element model to establish the normal foundation stresses resulting from the dam, temperature, and water loadings. These stresses are then used for the foundation sliding stability analysis (Meisenheimer 1992).

The shear-friction method of analysis applies to any section of the structure or its contact with the foundation. The shear-friction factor of safety, Q, is the ratio of resisting forces to driving forces as computed by the expression:

$$Q = \frac{CA + \left(\sum N + \sum U\right) \tan \phi}{\sum V} \tag{6}$$

where

C = unit cohesion

A = area of the section considered

ΣN = summation of normal forces

ΣU = summation of uplift forces

tan ϕ = coefficient of internal friction

ΣV = summation of shear forces

Equation (6) and symbols are listed as they appear in the Bureau of Reclamation (1976).

Selection of Design Parameters

Concrete properties

Values of concrete properties are estimated from published literature, earlier testing performed for the structures at the time of construction, or, if deemed necessary, samples obtained directly from the dam. If site-specific data are not available, the following average values for concrete properties (Bureau of Reclamation 1977) are used until test data are available for better results:

Compressive Strength - 3,000 to 5,000 psi
Tensile Strength - 5 to 6 percent of the compressive strength
Shear Strength:
  Cohesion - about 10 percent of the compressive strength
  Coefficient of internal friction - 1.0
Poisson's ration - 0.2
Instantaneous modulus of elasticity - 5.0×10^6 psi
Sustained modulus of elasticity - 3.0×10^6 psi
Coefficient of thermal expansion - 5.0×10^{-6}/°F
Unit weight - 150 pcf

Deformation modulus

Foundation deformations result from dam loading, which in turn affects the stress distribution within the dam and the overall stress distribution within the foundation. Initial studies generally use empirical relationships based on index properties obtained from drill core and site geologic mapping to estimate foundation modulus values. Should more accurate values be needed for 3-D finite element analysis, in situ field testing is generally performed, such as larger scale in situ jacking tests and cross-hole geophysical methods.

Shear strength parameters

Shear strength parameters for the concrete-to-rock bond and the intact rock are required for the sliding stability analysis. The parameters are generally conservatively assumed from available published data and the database available within the Bureau of Reclamation for initial analysis. Should the preliminary analysis indicate more refined parameters are required, then samples are obtained for laboratory triaxial and direct shear testing. Larger scale field direct shear tests are expensive and only performed if considered critical and warranted. The Coulomb method is used to determine the appropriate unit cohesion and the coefficient of internal friction.

Uplift

For dams without drainage curtains or nonfunctional drains, the uplift pressure is assumed to act across 100 percent of the foundation base and varies from full headwater pressure at the heel of the dam to full tailwater pressure at the toe of the dam. If a line of drains exist, the uplift pressure can be assumed to have a pressure at the drains equal to the tailwater pressure plus one-third the differential between the headwater and the tailwater. Full headwater pressure is still assumed at the heel of the dam and tailwater pressure at the toe. Geologic conditions and dam construction details, e.g., extended aprons, are considered when establishing the uplift pressure profile. If a history of detailed uplift monitoring data is available for the dam, this data will be used in lieu of the general assumptions.

Case History of Stability Analysis

Theodore Roosevelt Dam is an existing gravity-arch, masonry structure that will be raised 77 ft with conventional mass concrete. Extensive modifications to the dam and appurtenant structures are required and include outlet works and power penstock using a lake tap and tunnels, and a new spillway constructed within thrust blocks on each abutment.

Extensive and diverse geotechnical investigations were required to evaluate the proposed modifications to the Theodore Roosevelt Dam. Drainage adits were required to reduce uplift in order to meet stability requirements for the modifications. The adits were installed early so they could be used to obtain additional information needed to refine the design and analyses. The adits were used to (a) verify geologic conditions of the dam foundation, (b) determine drainage effectiveness through monitoring of piezometers and water levels, (c) examine the interior masonry from a short segment on the right side of the dam, (d) accurately locate the dam/foundation contact, and (e) test and sample the foundation rock and masonry. Details of the exploration testing analyses and design considerations are presented in the Bureau of Reclamation Geotechnical Summary Report (1990).

Deformation properties

Initial studies used empirical relationships based on rock index properties to estimate the foundation modulus values. Three different rock types (dolomite, sandstone, and shale) were present in the foundation, as well as a shear zone. Good foundation modulus information was considered important to assure the stress and load distribution from the 3-D finite analysis was appropriate. Later studies used cross-hole tomography, calibrated by means of large scale in situ jacking tests, to estimate modulus values across the foundation. The values determined by this later process were higher than those estimated by the empirical approach.

Shear strength parameters

The shear strength of joints, faults, clay-filled bedding partings, and intact rock forming bridges between discontinuous joints were estimated. The strengths were based primarily on laboratory direct shear tests with some field characterization included in the evaluation. An evaluation of shear strength versus shear deformation indicated that the full (peak) strength estimated for the joints and intact rock was compatible with displacements, and thus, the resistance provided by each could be added without reduction.

Water pressure distribution

The early installation of the drainage adits proved very valuable in collecting data on the effectiveness of the drainage system. Water levels within the foundation were monitored by piezometers, drill hole water levels, and seeps in the dam. Demonstrating that a very effective drainage system could be designed and installed was important in improving the stability in some critical areas to acceptable levels.

Stability analysis

Five critical foundation blocks were identified for rigid block limit equilibrium analysis. These blocks were bounded by joints and clay partings. For the static analysis, the following loads were applied to each wedge:

a. The weight of the wedge.

b. Water forces acting normal to the planes defining the wedge.

c. Static dam forces for each load case analyzed.

The compression forces on the rock foundation from the dam loading, which were used for the stability analysis, were determined from a 3-D finite element program SAPIV (Bathe 1973). A 3-D force resolution was performed to determine the potential sliding planes, the driving force, the normal and resisting forces acting on the potential sliding planes, and the resulting factors of safety for both the existing and the raised dam. The information gained from the field investigation studies, in conjunction with additional laboratory testing, provided the level of understanding necessary to more realistically model the project and have confidence in the results. Foundation block E was marginal as to meeting established factors of safety; however, due to the quality of the data and analyses, the Bureau of Reclamation and the Consultant Review Board considered the block stable and acceptable for the dam modification. All the summary information presented in this report was obtained from the Bureau of Reclamation (1990).

6 Tennessee Valley Authority Stability Design Criteria

General

The Tennessee Valley Authority (TVA) evaluates existing concrete gravity dams on an as-needed basis. Priorities for evaluating the stability of existing concrete gravity dams are determined by safety concerns or planned modification to the structures. The majority of all studies, investigation, instrumentation, analyses, and design are performed with in-house engineering staff and support contractors. All hydroelectric, dam, and river lock projects are reviewed by a TVA Hydro Consultants Board, which is composed of two to three internationally recognized experts. All methods of analyses, design parameters, conclusions, recommendations, and factors of safety are reviewed and concurred with by the Hydro Consultants Board prior to implementation by TVA. This review process, which is conducted twice a year or as required, provides for innovative approaches to solve particular problems when appropriate.

TVA has an internal guide, *Instructions for Review of TVA Dams and Appurtenances* (undated). The document provides the necessary guidelines and information for structural review of TVA dams to ensure conformance to the *Federal Guideline for Dam Safety*.

Design Load Cases

Typical sections of dams, spillways, powerhouse intakes, and locks that serve as primary water barriers are analyzed for the following static load condition: Case I-Headwater and tailwater for PMF. Analysis for Case I is initially done by the gravity method. Additional analyses will be performed should a more realistic approach be required. Priorities and the extent of analyses are based on review and compilation of project data, initial results, and the direction of the responsible engineer in charge.

A design instruction package is developed for each project that details instructions for the design and preparation of drawings and what standards

they will conform to. This design instruction package will include pertinent design criteria and parameters considered applicable. The instructions will also outline specific design load cases for each structure. Special considerations include such items as headwater and tailwater elevations, uplift considerations, foundation criteria, and other pertinent design information for each case that needs to be analyzed.

Stability Criteria

The safety factor against sliding was recently evaluated for a concrete dam using a conventional 2-D sliding stability analysis by the Bureau of Reclamation (1976). The stability criteria used in the evaluation are a slightly modified version of those used in EM 1110-2-2200 (USACE 1958). According to the guidelines, the dam is stable if at least one of the following criteria is met.

a. Test 1: The sliding factor.

$$\Sigma F_h / \Sigma F_v \leq \tan \phi / 1.5 \tag{7}$$

ΣF_v and ΣF_h are the sums of the vertical and horizontal forces in pounds, ϕ is the friction angle, and *1.5* is a safety factor.

b. Test 2: The safety factor $Q > 4.0$ where Q is calculated.

$$Q = \frac{\Sigma F_v \tan \phi + CA}{\Sigma F_h} \tag{8}$$

In the equation, ΣF_v, ΣF_h, and ϕ are the same as before, C is the cohesive strength in psf, and A is the area of the uncracked part of the sliding surface in square feet.

Selection of Design Parameters

Concrete properties

Concrete properties are fairly well-established within TVA unless site-specific data are available. Table 6 provides the recommended properties for concrete to be used in the analysis.

Shear strength parameters

The allowable shear strength for intact concrete is 400 psi. The friction angle (ϕ) is assumed to be 33 deg for both the concrete and rock. The

allowable shear strength for the rock-to-concrete interface is 250 psi where the interface is considered intact and rock quality is considered good. If the concrete or rock interface is cracked, then sliding is resisted only by the friction angle. Weak seams, such as fault seams, clay, or clay shale, are handled separately, as required.

Uplift

The assumptions to be used in analysis for uplift are presented in Table 7.

Allowable stresses

The allowable stresses to be used in evaluating stability analyses are presented in Table 8.

7 Comparison of U.S. Army Corps of Engineers Criteria with Other Methods

Design Load Cases

Design load cases and the actual static design load condition classification for the CE, FERC, Bureau of Reclamation, and TVA are compared in Table 9. Design load conditions for EM 1110-2-2200 (USACE 1958) and revised EM 1110-2-2200 (USACE 1994a) are shown for comparison. Details of each of these load conditions are presented in the representative sections of this report for each design agency. Overall, there is little difference in the design load case assumptions used by each agency for the normal operating level and flood loading design conditions. Specific design loads to be included in the analysis such as dead load, silt load, and ice pressure are also similar for each of the design load conditions presented. The CE does not have any design load cases that are considered unusual when compared with the remainder of the industry standards for analysis of existing concrete gravity dams.

Factors of Safety

The factors of safety used to evaluate the sliding stability for concrete gravity dams are compared in Table 10. Comparison is made to the CE factor of safety criteria presented in the revised EM 1110-2-2200 (USACE 1994a) since this is essentially the CE acceptance criteria at the present time. The factor of safety criteria presented in Table 10 represent the requirements for new design, as well as the requirements for reevaluating existing concrete gravity dams for new design load assumptions. All of the design agency criteria presented allow special consideration for calculated factors of safety when reevaluating existing dams since historical loadings, performance, monitoring data, and general conditions provide information on the conservatism in the analysis that may be biasing the results of the analysis.

The factors of safety presented for each design agency do vary somewhat as shown in Table 10. The CE and FERC guidelines have been recently updated and reflect a more detailed design guidance criteria for assessing concrete dam stability. These newer design guidelines reflect recent advances in understanding the performance of existing concrete gravity dams, reflect foundation interaction that occurs, and allow credit to be taken when performance data and specific design criteria are available or can be obtained. A more realistic approach to stability analysis also allows for a less conservative factor of safety to be required when determining acceptability.

For all of the agencies, a factor of safety lower than the recommended criteria can be utilized when evaluating existing dams. Calculated factors of safety are evaluated on a case-by-case basis for each project. A lower factor of safety is sometimes allowed for acceptance of the stability analysis when detailed site information is available to define and support a more realistic sliding analysis. This includes an appropriate definition of the failure surface and failure mechanisms and high confidence in critical design parameters such as uplift distribution and the shear strength parameters (cohesion and angle of internal friction) for the failure surface.

Having the detailed information to perform a more refined and appropriate analysis also allows for a better definition of conservatism that still remains in the analysis. Appropriate review authorities within an agency, or their consultant boards, need this type of data and analysis base to allow for relaxation of the factor of safety criteria and still have confidence that the analysis is conservative and represents the existing dam as safe for the design loadings.

The factors of safety for sliding stability in the revised EM 1110-2-2220 (USACE 1994a) are in line but clearly lower than current values being used by other agencies to evaluate safety for existing dams. The CE factors of safety are also in line with those being accepted by other prestigious foreign agencies. B.C. Hydro (Cornish and Moore 1991) accepted factors of safety of 1.5, 1.3, and 1.1 for usual, unusual, and extreme loading conditions, respectively, after reassessing several existing dams. Sampling and shear testing methods were developed and used extensively for reassessing the strength of bedding planes underlying the dams. Foundation reassessments for B.C. Hydro have followed these more recent procedures.

Methods of Analyses

Overturning

The CE and TVA perform overturning analysis for concrete gravity dams and lock structures in a similar manner to locate the resultant and to define areas of nonecompression and base contact. The Bureau of Reclamation and FERC require a cracked base analysis for existing dams using the procedures outlined in the design manuals (Bureau of Reclamation 1976, 1977), as

described previously in this report. This analysis is performed to determine whether cracking has occurred in the foundation.

The results from both types of analyses are used to define areas of non-compression or tension in the foundation base and to determine the area of the dam that is considered in contact with the rock. These assumed contact and non-contact areas are then utilized to define uplift conditions and shear resistance along the base for further sliding stability analysis. Overturning acceptability and allowable compression in the concrete and foundation are also determined. Even though the methods of analyses for overturning and cracked base are somewhat different, the results achieved and definitions for contact and non-contact foundation base areas are fairly consistent.

Stability analysis

The sliding stability analysis method preferred by the CE is the limit equilibrium method. This is also the sliding stability analysis method in the FERC Report (1991), and it is presently the method preferred by the Bureau of Reclamation, even though it was not referenced in the Bureau of Reclamation design manuals (1976, 1977).

Currently, approaches used by the CE for performing stability analyses of existing dams are consistent with methods used by the dam industry. Technical conferences, agency interaction for transfer of technical and performance experience, and national programs such as the EPRI Concrete Gravity Dam Stability Program are providing opportunities to transfer technology and experience. This transfer of information is providing better analytical tools and experience so that more innovative and less conservative approaches can be used for stability analysis. This is reflected in recent revisions to FERC and CE guidelines.

Using high speed PC's and new software programs allow for parametric evaluations to be performed quickly and cost effectively. The sensitivity of input parameters to the stability analysis can be determined and evaluated. The sensitivity of various assumed failure planes can be assessed quickly to establish the zones critical to dam stability. This type of sensitivity analysis is beneficial to establish a level of confidence for the input parameters and failure surfaces being used in the stability analysis. Also, it can highlight and justify where site-specific information should be obtained to refine the analyses, such as exploration programs to better define uplift, shear strength, or failure planes.

The tools for reasonable stability analysis are available and improvements are continuing to be made. Two-dimensional stability analysis is acceptable for most conditions encountered for concrete gravity dams. All of the guidelines and present engineering practices by the CE, FERC, Bureau of Reclamation, and TVA allow for more appropriate methods of analysis to be performed when justified by specific site conditions. The differences seen in analytical results today have more to do with selecting appropriate uplift and

shear strength parameters rather than selecting the appropriate method of analysis.

Selection of Design Parameters

Uplift

Uplift pressure distribution is a very sensitive parameter for stability analysis. Table 11 compares uplift assumptions recommended by each of the design agency guidelines. Uplift assumptions are fairly consistent between each guideline. For existing dams, the guidelines all allow for more appropriate uplift assumptions to be used when supported by uplift monitoring data at the specific dams. Since many older dams do not have uplift monitoring data, conservative assumptions will have to be utilized. Parametric analysis can be utilized to determine whether it is justified to install piezometers to establish an uplift distribution for the particular dam being analyzed. The CE guidelines for determining uplift beneath the dam are consistent with industry practice.

Shear strength parameters

The results of the stability analysis are extremely sensitive to the selection of the failure surface, mode of failure along the surface, and the selection of shear strength parameters C and ϕ. Selecting failure surfaces and shear strength parameters should be more than just a published literature review; it should incorporate all known site information with the experience and judgment of a knowledgeable geotechnical or geological engineer. The failure surfaces will be defined based on the best geologic information available for the dam foundation. Shear strength parameters generally have to be conservative for initial stability analysis since most existing dams have very limited geotechnical data. This approach to the initial analyses is acceptable and standard. Parametric analysis can be performed to evaluate the sensitivity of the shear strength parameters in order to define whether further exploration and testing are justified and potentially beneficial to refine design input parameters. A problem in selecting initial shear strength design parameters is that published data are not well consolidated and readily available, so very conservative parameters are generally used. This can considerably bias the results toward a very conservative analyses and remedial fix. Some dam owners at present do not elect to spend money for site exploration programs and accept an expensive fix.

EM 1110-1-2908 (USACE 1994a) and Nicholson (1983) provide a comprehensive guide for rock mass characterization and selecting shear strength parameters. Recommendations and references are provided for performing field investigations, laboratory testing, rock mass classification, rock failure characteristics, and selection of shear strength parameters. The above references are considered to be excellent resources and are examples where the CE has made a major contribution to industry standards and practice.

Summary

With the issuance of the revised EM 1110-2-2200 (USACE 1994a) and EM 1110-1-2908 (USACE 1994b), the CE has established the process for selection of input criteria and guidelines for analyses that are consistent and up-to-date with other design agencies and industry practice for evaluating existing dams. These two Engineering Manuals also address some of the questions raised by CE districts concerning the many CE documents presently required to understand and perform stability analyses for existing concrete gravity dams and structures (USACE 1986). The CE has provided tools and guidance and is up-to-date in their approach for sliding stability analysis. It is still apparent that the selection of input parameters and the judgment and flexibility allowed by the criteria are not consistently understood throughout CE districts. It is again emphasized that proper recognition and understanding of the geologic features within the dam foundation are required. The geologic features that affect the performance and behavior of an existing dam must be clearly defined so that failure mechanisms, uplift, and shear strength parameters can be properly selected. It is only at this point in time that analytical techniques can be properly tailored to fit the dam and foundation conditions being evaluated for stability.

8 Factors Significant for Stability Analysis of Existing Concrete Gravity Dams

Conservatism in Present Analysis

Methods of analysis

The limit equilibrium method of stability analysis assumes planar failure surfaces, as do most of the 2-D methods of analyses. This assumption in many cases introduces considerable conservatism in that the potential failure surfaces can be quite irregular, resulting in direct shearing of rock as well as slippage along defined surfaces. This can be somewhat accounted for in the selection of shear strength design parameters, but still remains an engineering judgment that must remain on the conservative side. Also, interaction with adjacent wedges is neglected. Any movement within the dam foundation is very much as 3-D interaction and this will generally result in additional sliding resistance that is not accounted for in the analysis. This known conservatism, which is inherent to the analysis, can be given credit and be accounted for when an existing dam is just under the acceptable criteria for stability.

Uplift

Uplift assumptions used for the CE and various agency design criteria are very consistent. Areas where the recommended uplift design assumptions can still be very conservative and restrictive are related to the assumptions made for full headwater in the noncompression area. The general assumption used by the dam industry for years is that a crack may have developed, so full uplift is to be expected.

For most dams, the foundation rock exists in a fractured condition and fracture flow beneath the dam is occurring as a result of the headwater pressure. Since the theoretical crack at the base is generally interconnected with existing fractures in the rock, these fractures can dissipate high pressure

changes that are assumed in a cracked base analysis. Also, with the effective stress changes that occur at the base of the dam during extreme loadings and the flexibility of the foundation, a theoretical crack may not develop as analyzed and the assumption of full uplift throughout the base crack is probably not a reality. This characteristic was observed in the EPRI (1992) study. The actual uplift profile for high headwater loads may be considerably less than what is recommended for use by the guidelines.

This same reasoning applies to the requirement to disregard drain effectiveness should the noncompression area move beyond the drain line. Again, observed uplift performance during the EPRI study did not support this design recommendation in all cases. When this uplift design assumption severely penalizes the analysis, further evaluation and study may be justified and should not be ignored.

Selection of shear strength parameters

The selection of appropriate C and ϕ values to be representative of the assumed failure surfaces and modes is probably one of the more difficult tasks. The outcome of the analysis has a high sensitivity of these input parameters. Considerable data are available, but through many sources. The data are not consolidated and readily obtainable. The EPRI report (1992) and a CE database at WES have made a major effort to compile representative C and ϕ values. These databases are available to the CE and should be utilized. It should be emphasized that one must be extremely careful in using strength values from various databases. Many factors, including variations in rock types, sample selection, testing techniques, and effective normal stress levels have to be considered when trying to make a proper selection of representative parameters. If values from various database are to be used, e.g., for preliminary analyses in the absence of site-specific data, efforts should be made to select values from sites in similar geologic regions or settings where rocks (or rock discontinuities) of similar geologic characteristics were tested under appropriate normal stress levels.

Selecting appropriate design parameters, even when the data are available, requires considerable experience and engineering judgment. When site-specific information is limited, C and ϕ values must be conservatively selected. In many cases, the lowest C and ϕ values from test results are defined as governing without understanding why they should be considered representative. When the experience level in selecting rock strength parameters is limited, it is generally advisable to obtain some appropriately experienced assistance, either from within the organization or from external consultants, in order to avoid misinterpreting the database.

Another factor in determining C and ϕ is that test results are obtained from an available sample size (generally 6-in. diam or less). Test equipment capability, sample preparation, and test procedures utilized have to be considered when evaluating test results. The size effect from laboratory testing to field

conditions is not always well understood. Conservatism must be adjusted based on the reliability and confidence of the data available.

Areas of Improvement

Training

Training is considered one of the major items that would result in the most beneficial and quickest improvement to the CE. With the issuance of EM 1110-2-2200 (USACE 1994a) and EM 1110-1-2908 (USACE 1994b), the evaluation approach, design criteria, and parameter selection criteria have been consolidated into two major documents. It is apparent through discussions with CE staff that confusion exists concerning how to analyze existing concrete gravity dams. It is unclear what options are allowed by the design criteria and what are the appropriate methods and basis for selecting failure surfaces and critical design parameters.

With the issue of the new design manuals, serious consideration should be given to conducting workshops for affected design engineers from throughout the CE. Consistency in the approach for analysis of existing dams will provide for a definite measure of improvement.

CE geotechnical staff and structural staff should be involved in the reevaluation of existing dams. The analysis of stability for an existing dam should be performed by a structural and geotechnical team. In addition, the experience level that exists within certain districts, divisions, and at WES could be utilized more effectively in information transfer and technical support.

Joint training workshops with the structural and geotechnical teams within a district and with other districts, divisions, and WES will assist in experience exchange and technology transfer. It will also help establish networking contacts that can be used in joint problem solving and technical assistance.

Other areas of training that could be addressed and could provide immediate results include:

a. Developing a strategy for approaching the evaluation of existing concrete gravity dams and structures on rock.

b. Performing initial stability analysis and defining parameters sensitive to the analysis.

c. Understanding uplift and selecting appropriate design criteria. The EPRI report (1992) could be used as training basis.

d. Developing project case histories that are in conformance with the new design standards and that can be used by the CE as an example for training. These case histories should include the field exploration

program and testing, the selection of failure modes, and the selection of C and ϕ input parameters.

No significant improvements to the CE design criteria to be issued in 1994 were identified at this time. Justification for improving the selection of less conservative design parameters needs additional investigation.

Recommendations

Uplift

A better definition of the appropriateness of the uplift design assumptions to be used for noncompression areas could be beneficial for both concrete gravity dams and lock structures. The EPRI Report (1992) has an extensive uplift profile database. Additional databases were found to exist for CE projects, which were not incorporated into the EPRI study. These uplift databases can be reviewed with respect to noncompression and compression areas beneath a dam to better define actual uplift response for known geologic conditions and foundation treatment.

Uplift pressure with corresponding headwater and tailwater changes need to be monitored, particularly when analysis indicates an area is in noncompression. Piezometers need to be installed in the noncompression zone if data are not available. Also, it is recommended that extensiometers be installed and monitored to measure foundation and gravity structure deflection with headwater and uplift pressure changes.

A navigation lock would be an excellent model for evaluating this condition since drainage galleries are not present. A gravity dam that has a drain gallery close to the upstream face would also be a good condition to evaluate the effectiveness of drainage.

Shear strength parameters

A CE database should be developed and continually updated for use as a resource for obtaining input parameters for preliminary assessments. This database would also be useful to correlate and evaluate laboratory testing results. A good database, in which the quality of sampling and testing is known, can assist in minimizing testing required and improve the confidence level in selecting more appropriate design criteria from these results. It should be emphasized that one must be extremely careful in using strength values from various databases.

A better definition needs to be developed for the concrete-to-rock bond. The cohesion of bonding that occurs at the foundation contact is generally ignored in the analysis. The EPRI (1992) tests defined bond strength fairly

well. The CE needs a method to allow for concrete-to-rock bonding, where it is known to occur and where it could be beneficial to the analyses.

Research is needed to better define the fundamentals of stress and strain parameters for irregular rock surfaces so that more appropriate C and ϕ values can be used in analyses. A better understanding of stress and strain parameters for various rock materials would result in a higher confidence level in selecting appropriate C and ϕ values for the various failure mechanisms expected.

This research could be used to better define shear strength input parameters that might be used for irregular shear surfaces, which are considered planar in stability analyses. A combination of laboratory and larger scale field tests are required for this comparison. The large centrifuge being built at WES could be used for this type of research program, by analyzing stability problems involving irregular shear surfaces within the rock and at the base of the concrete dam. Also, the combined effects of weak and competent foundation zones beneath a dam could be evaluated.

Foundation geology definition

The characteristics of foundation geology are difficult to determine for many existing concrete gravity dams. Uplift is directly related to the joint and fracture characteristics of the foundation geology, since it is controlled primarily by fracture flow. Foundation modulus and dam deflection are also influenced by foundation geology, as well as potential failure planes that might develop. The tightness of joints and fractures also influence the stress-strain behavior, which can in turn be a consideration in selecting C and ϕ design parameters. Tightness is also a consideration for the 3-D interaction that would happen should sliding movement occur.

It is recommended that a Rock Mass Rating (RMR), such as presented in EM 1110-1-2908 (USACE 1994b) or other rock mass classification systems, be evaluated with respect to better defining foundation characteristics from limited boring data, borehole camera logging, and correlation with surface mapping. The rock mass classification system should also be assessed with the critical input parameter assumptions used for stability analysis, to determine if it can be used as a tool to improve the confidence level in parameter selection. A rock mass classification system does not replace site investigations and geologic assessment. They are adjunct to these items and only have a special value for relating the rock mass in question to good engineering judgment.

9 Developing a Site Investigation Strategy for Existing Concrete Gravity Dams

Prior to developing a site investigation strategy, an initial overturning and sliding stability analysis should be performed to assess the current condition of the structure. This analysis can be performed using the basic design assumptions and examples presented in EM 1110-2-2200 (USACE 1994a) and EM 1110-1-2908 (USACE 1994b). Based on available geologic information, conservative assumptions can be used to define an assumed failure surface, uplift, and shear strength input parameters. Parametric analysis can be performed, as required, to define the sensitivity of the parameters. A field exploration and testing program must be developed to obtain the data necessary to refine uplift or shear strength parameters enough to significantly improve the reliability of the stability analysis.

A second consideration prior to implementing a detailed field exploration program would be to compare the cost of the required detailed site program to the estimated remedial costs saving that might be expected with a refined stability analysis. Field exploration programs generally have to be conducted to define parameters for remedial design, so these costs have to be factored into a cost benefit analysis. The cost benefit of performing field investigations help define the level of work that is justified. The exploration program can be then finalized and implemented.

Preparing an exploration and testing program is usually defined by the initial stability analysis performed and focused where field geologic information, laboratory testing, and monitoring will refine the input to the analysis such that it is more representative and reflects more accurately the stability of the structure. While components of a site investigation program differ somewhat, the program's goal is to provide a 3-D description of the site geology and to define unique characteristics that are considered critical to an appropriate analysis. The field program should also include collecting design criteria that may be required for remediation alternatives.

Key elements that should be defined include:

a. Presence of faults, shear zones, or weak seams.

b. Representative uplift pressure distribution.

c. Dam/foundation contact characteristics.

d. Foundation rock mass characteristics.

e. Sample collection required for testing.

f. Instrument selection and installation.

g. Surface mapping around dam.

h. Dam inspection for condition and performance characteristics.

Details and practical guidelines for conducting field investigations, sample collection and handling, and laboratory testing are described in EPRI (1989) and EM 1110-1-2908 (USACE 1994b). Drilling and sampling success depends upon using proper procedures and recovering representative cores from the locations desired. Samples must be carefully handled and protected during recovery, shipment, and testing in order to obtain meaningful results. In order for data from field instrumentation to be worthwhile, the instrumentation must be carefully installed and monitored with all pertinent conditions recorded.

A methodology for reevaluating existing dams is presented in Chapter 8 of EM 1110-2-2200 (USACE 1994a). Chapter 14 of the EPRI report (1992) also presents steps required to evaluate stability and incorporate site-specific foundation and uplift data. Adequate information is available within these two documents to develop a training package to assist engineers that are reevaluating existing concrete gravity dams and structures.

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Table 1
U.S. Army Corps of Engineers Design Stability and Stress Criteria

Load Condition	Resultant Location at Base	Minimum Sliding FS[1]	Foundation Bearing Pressure	Concrete Stress[2]	
				Compressive	Tensile
Usual	Middle third	2.0	≤ Allowable	$0.3\ f_c$	0
Unusual (SPF)	Middle half	1.7	≤ Allowable	$0.5\ F_c$	$0.6\ f_c^{2/3}$
Extreme (PMF)	Within base	1.3	≤ 1.33 × Allowable	$0.9\ f_c$	$1.5\ f_c^{2/3}$

[1] The sliding factor of safety (FS) is based on a comprehensive field investigation and testing program.
[2] f_c is the 1-year unconfined compressive strength of concrete.
Source: EM 1110-2-200 (USACE 1994).

Table 2
EPRI Concrete Gravity Dam Stability Study Host Dam Summary

Hot Dam Summary										
Dam	Owner	State	River	Year Completed	Max Height, ft	Crest Length, ft	Foundation Rock	Grout Curtain	Drain Curtain	EPRI Program
1. Carpenter	Arkansas Power and Light Co.	Arkansas	Ouachita	1932	118	1,160	Sandstone and Shale	Yes	Yes	Uplift and strength data
2. Cochrane	Montana Power Co.	Montana	Missouri	1958	103	869	Sandstone, Shale, and Siltstone	Yes	Yes	Core drilling, strength testing, uplift data
3. Douglas	TVA	Tennessee	French Broad	1943	203	1,706	Dolomite	Yes	Yes	Frequent uplift readings
4. Fontana	TVA	North Carolina	Little Tennessee	1944	482	2,365	Quartzite and Phyllite	Yes	Yes	Frequent uplift readings
5. Friant	USBR	California	San Joaquin	1942	319	3,488	Quartz-biotite Schist	Yes	Yes	Frequent uplift readings
6. Grand Coulee Forebay	USBR	Washington	Columbia	1974	201	1,170	Granite	Yes	Yes	Frequent uplift readings
7. Hungry Horse (Thick Arch)	USBR	Montana	Flathead, South Fork	1953	564	2,115	Limestone	Yes	Yes	Frequent uplift readings
8. Lloyd Shoals	Georgia Power Co.	Georgia	Ocmulgee	1912	100	1,070	Granite and Granite Gneiss	No	No	Core drilling, strength testing, uplift data
9. Mahoning	USACE	Pennsylvania	Mahoning Creek	1941	162	926	Shale, Siltstone and Sandstone	Yes	No[1]	Uplift and strength data
10. Morony	Montana Power Co.	Montana	Missouri	1930	94	883	Sandstone and Shale	Yes	Yes	Core drilling, strength testing, uplift data
11. Norris	TVA	Tennessee	Clinch	1936	265	1,860	Limestone and Dolomite	Yes	Yes	Frequent uplift readings
12. Pickwick Landing Lock	TVA	Tennessee	Tennessee	1984	94	1,000	Limestone and Shale	Yes	No	Frequent uplift readings during lock operation

(Continued)

[1] Drains attached to header.
Source: EPRI (1992).

Table 2 (Concluded)

Hot Dam Summary (Continued)										
Dam	Owner	State	River	Year Completed	Max Height ft	Crest Length ft	Foundation Rock	Grout Curtain	Drain Curtain	EPRI Program
13. Pit 6	Pacific Gas and Electric Co.	California	Pit	1965	224	582	Volcanic Flows and Pyroclastics	Yes	Yes	Core drilling, strength testing, uplift data, drain cleaning data
14. Pit 7	Pacific Gas and Electric Co.	California	Pit	1965	248	907	Meta-sandstone	Yes	Yes	Core drilling, strength testing, uplift data, drain cleaning data
15. Scott	Pacific Gas and Electric Co.	California	Eel	1921	134	805	Sandstone and Shale	Yes	Yes	Uplift data
16. Shaver Lake	Southern California Edison Co.	California	San Joaquin	1927	185	1,760	Granite	Yes	Yes	Frequent uplift readings
17. Upper Baker	Puget Sound Power and Light Co.	Washington	Baker	1959	312	1,231	Phyllite and Hornfels	Yes	Yes	Core drilling, strength testing, uplift data, piezometer installation, drain cleaning data

Table 3
Concrete-to-Rock Contact Peak Shear Strength Summary

Contact Rock Type	Number of Shear Tests Recorded	Number of Tensile Tests Recorded	Best Fit Values			Lower Bound Values	
			Cohesion, psi	Friction Angle, deg	Correlation Coefficient	Cohesion, psi	Friction Angle, deg
Granite	6	11	183	54	0.84	95	53
Granite-Gneiss	4	6	189	57	0.87	70	57
Greenstone	2	0	No line drawn	--	--	--	--
Limestone/Dolomite	9	0	278	68	0.49	165	68
Phyllite	3	1	240	62	0.84	70	62
Sandstone	15	5	260	65	0.80	50	65
Shale (Laboratory)	9	0	17	60	0.79	0	48

Source: Electric Power Research Institute (1992).

Table 4
Concrete-to-Rock Residual Shear Strength Summary

Contact Rock Type	Number of Tests	Best Fit Values			Lower Bound Values		Sample Types Tested
		Cohesion C, psi	Friction Angle, deg	Correlation Coefficient	Cohesion, psi	Friction Angle, deg	
Granite	6	11	35	0.93	0	32	Natural
Granite-Gneiss	4	4	34	0.99	0	31	Natural
Greenstone	2	0	37	--	--	--	Natural
Limestone/Dolomite	12	17	35	0.58	0	23	Saw cut, cast
Phyllite	5	0	39	0.89	--	--	Natural
Sandstone	46	25	29	0.6	0	27	Natural, saw cut, cast
Shale (Laboratory)	13	0	34	0.75	0	13	Natural, saw cut, cast
Siltstone	13	15	24	0.83	0	22	Natural, saw cut

Source: Electric Power Research Institute (1992).

Table 5
FERC Recommended Factors of Safety[1]

Dams having a High or Significant Hazard Potential	
Loading Condition	**Factor or Safety[2]**
Usual Unusual	3.0 2.0
Dam having a Low Hazard Potential	
Usual Unusual	2.0 1.25

[1] Factors of safety apply to calculating stress and the shear friction factor of safety within the structure, at the rock-to-concrete interface, and in the foundation.
[2] Factors of safety are based upon using the gravity method of analysis.
Source: Federal Energy Regulatory Commission (1991).

Table 6
Tennessee Valley Authority Design Criteria for Concrete Properties

Concrete Data	
Weight (for stability analysis): Intake, powerhouse, and service bay Spillway dam, non-overflow dam and locks	 - 145 lb/cu ft - 150 lb/cu ft
Weight (for structural design): All elements	- 150 lb/cu ft

Strength

Type	Compressive Strength, f'c, psi	Modulus of Elasticity, E, psi	Modulus of Rigidity G, psi
Mass, Un-reinforced	2,000	2,500,000	1,000,000
3-in. face and reinforced	3,000	3,200,000	1,300,000
1-1.5 in. face and reinforced	3,000	3,2000,000	1,300,000

Coefficient of thermal expansion - 0.000006

Note: The data above are for general use. If data compiled from original project reviews are of less magnitude, the original project data shall be used. If inspection records have indicated problem areas with regard to structural deficiencies and/or to material properties, testing may be required to establish strength parameters.
Source: Tennessee Valley Authority (Undated).

Table 7
Tennessee Valley Authority Uplift Design Considerations

	Upstream Face	Downstream At Drains	Downstream Face
Sections with drains	HW	TW + 1/4 (HW-TW)	TW
Sections without drains	HW	-	TW
Locks	TW + 2/3 (HW-TW)	-	TW

Intensity varies uniformly between points. Pressure acts over 100 percent of the base area.

If reviews for a project indicate uplift data exceeding the above assumptions, that uplift shall be used for the project evaluation.

Source: Tennessee Valley Authority (Undated).

Table 8
Tennessee Valley Authority Allowable Stresses

Allowable Compression, psi			Allowable Tension, psi	
Case	Rock Base	Concrete Base	Rock Base	Concrete Base
I	1,000	0.5 f'c	By Engineer in charge	100

For planes of analyses at concrete/rock contact, the resultant of all loads in Case I shall fall within the base.

Special instructions will be given by the responsible engineer in charge to evaluate results of dynamic analyses if such analyses are required.

Source: Tennessee Valley Authority (Undated).

Table 9
Comparison of Static Design Load Conditions and Design Load Condition Classification

Design Load Case	Corps of Engineers 1958	Corps of Engineers 1994	Federal Energy Regulatory Commission	Bureau of Reclamation	Tennessee Valley Authority
1. Normal Operating Level					
Headwater at operating pool or top of spillway gates. Tailwater at normal or minimum level.	Load Condition II	Usual	Case I Usual	Usual	--
2. Flood Loading to Include PMF					
Headwater at maximum floodpool elevation. Tailwater at appropriate flood elevation for spillage rate to give most critical head differential loading on dam.	Load Condition III	Unusual	Case II Unusual	Unusual	Case I
Tailwater elevation at full spillage rate through spill-ways and overtopping, occurring at PMF.	Load Condition IV	Extreme	Case II Unusual	Unusual	Case I

Table 10
Factors of Safety for Sliding Stability Concrete Gravity Dams

Load Condition	Bureau of Reclamation (1977)	Federal Energy Regulatory Commission (1991)	Corps of Engineers (1994)	Tennessee Valley Authority
Usual (Normal Pool)	4.0[e] 3.0[f]	3.0[a] 2.0[b]	2.0[g]	--
Unusual/Extreme (Flood Load - PMF or Max. Design Level unless noted)	2.7[e] 2.0[f]	2.0[a] 1.25[b]	1.7[g,h] 1.3[g]	4.0[c] 1.5[d]

[a] High or significant hazard dam.
[b] Dam having low hazard potential.
[c] When $C>0$ and ϕ is used.
[d] When $C=0$ and only ϕ is used.
[e] Sliding on any plane of weakness within the foundation.
[f] For concrete-to-rock contact.
[g] Using limit-equilibrium stability analysis method.
[h] Standard project flood (SPF).

Table 11
Uplift Criteria for Evaluating Existing Concrete Gravity Dams Uplift Assumptions at Base of Existing Dam

Uplift Assumption	Corps of Engineers	Federal Energy Regulatory Commission	Bureau of Reclamation	Tennessee Valley Authority
1. No drainage curtain or clogged drains	HW to TW	HW to TW	HW to TW	HW to TW
2. Drainage curtain	HW to drain to TW (drain efficiency 25 percent to 50 percent)[1]	HW to drain to TW (drain based on efficiency "E")[2]	HW to drain to TW (33 percent efficient at drain)[2]	HW to drain to TW (drain efficiency 25 percent)[2]
3. Crack at base - noncompression zone at base with no drainage	100 percent HW thru zone, then HW to TW	100 percent HW thru zone,[3] then HW to TW	100 percent HW thru zone,[3] then HW to TW	100 percent HW thru zone, then HW to TW
4. Noncompression zone short of drains	100 percent HW thru zone, then HW to drain to TW (drain efficiency considered)	100 percent HW thru zone, then HW to drain to TW (drain efficiency considered)	100 percent HW thru zone, then HW to drain to TW (drain efficiency considered)	100 percent HW thru zone, then HW to drain to TW (drain efficiency considered)
5. Noncompression zone thru drainage curtain	100 percent HW thru zone, then HW to TW (drain efficiency not considered)[4]	100 percent HW thru zone, then HW to TW (drain efficiency not considered[3]	100 percent HW thru zone, then HW to TW (drain efficiency not considered)[3]	100 percent HW thru zone, uplift then based on expected drain efficiency[3]

Notes:
HW - Headwater elevation at heel of dam, or end of noncompression zone.
TW - Tailwater elevation at toe of dam.
Drain - Elevation uplift at drain line based on drain efficiency or measured uplift.
[1] When foundation testing and uplift monitoring provide support justification, drain effectiveness can be increased to a maximum of 67 with CECW-ED approval.
[2] Drain efficiency based on measured uplift data. Percent of drain efficiency may be increased when field uplift monitoring data and drain performance provide additional substantiation for increase.
[3] Reduction allowed if performance of dam and measured uplift data support using reduced uplift assumptions in noncompression zone.
[4] The design office can request from CECW-ED a deviation if extensive remedial measures are required to satisfy this loading assumption for existing dams.

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The U.S. Army Corps of Engineers is presently reanalyzing the stability of many of their concrete gravity dams due to new flood loading design requirements. Past practice has been to use to same stability criteria for reviewing existing structures as is used for designing new structures. Using the existing U.S. Army Corps of Engineer guidelines and conservative design parameters for cohesion, shear strength, and uplift, many of these dams have been determined to be unstable under the new design loads and will require expensive rehabilitation. A more realistic analysis of the stability of many of these dams could result in considerable cost savings for rehabilitation.

The reanalysis of existing dams due to new flood loading requirements is not only an issue with the U.S. Army Corps of Engineers but also affects all federally regulated dams in the United States as well as dams of the Bureau of Reclamation and the Tennessee Valley Authority. Much work has been performed collectively through the Electric Power Research Institute by the dam industry to address some issues that are critical and sensitive to the stability analysis of concrete gravity dams.

(Continued)

14. SUBJECT TERMS
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This report summarizes the present accepted design practices used by Federal agencies in the United States and provides a comparison with the U.S. Army Corps of Engineers' new proposed guidelines, EM 1110-2-2200 "Gravity Dam Design" and EM 1110-1-2908 "Rock Foundations," which will be issued late this year. Also, recommendations are provided where additional focus needs to be applied to activities that can be beneficial to the Corps of Engineers in reevaluating the stability of existing concrete gravity dams.